Technical Report REMR-GT-21
October 1993

US Army Corps
of Engineers
Waterways Experiment
Station

Repair, Evaluation, Maintenance, and Rehabilitation Research Program

Incorporation of Wall Movement and Vertical Wall Friction in the Analysis of Rigid Concrete Structures on Rock Foundations

by Shannon and Wilson, Inc.
Engineering and Applied Geosciences

WES

Approved For Public Release; Distribution Is Unlimited

Prepared for Headquarters, U.S. Army Corps of Engineers

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	Problem Area		Problem Area
CS	Concrete and Steel Structures	EM	Electrical and Mechanical
GT	Geotechnical	EI	Environmental Impacts
HY	Hydraulics	OM	Operations Management
CO	Coastal		

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St. Louis, MO 63141-7126

Final report

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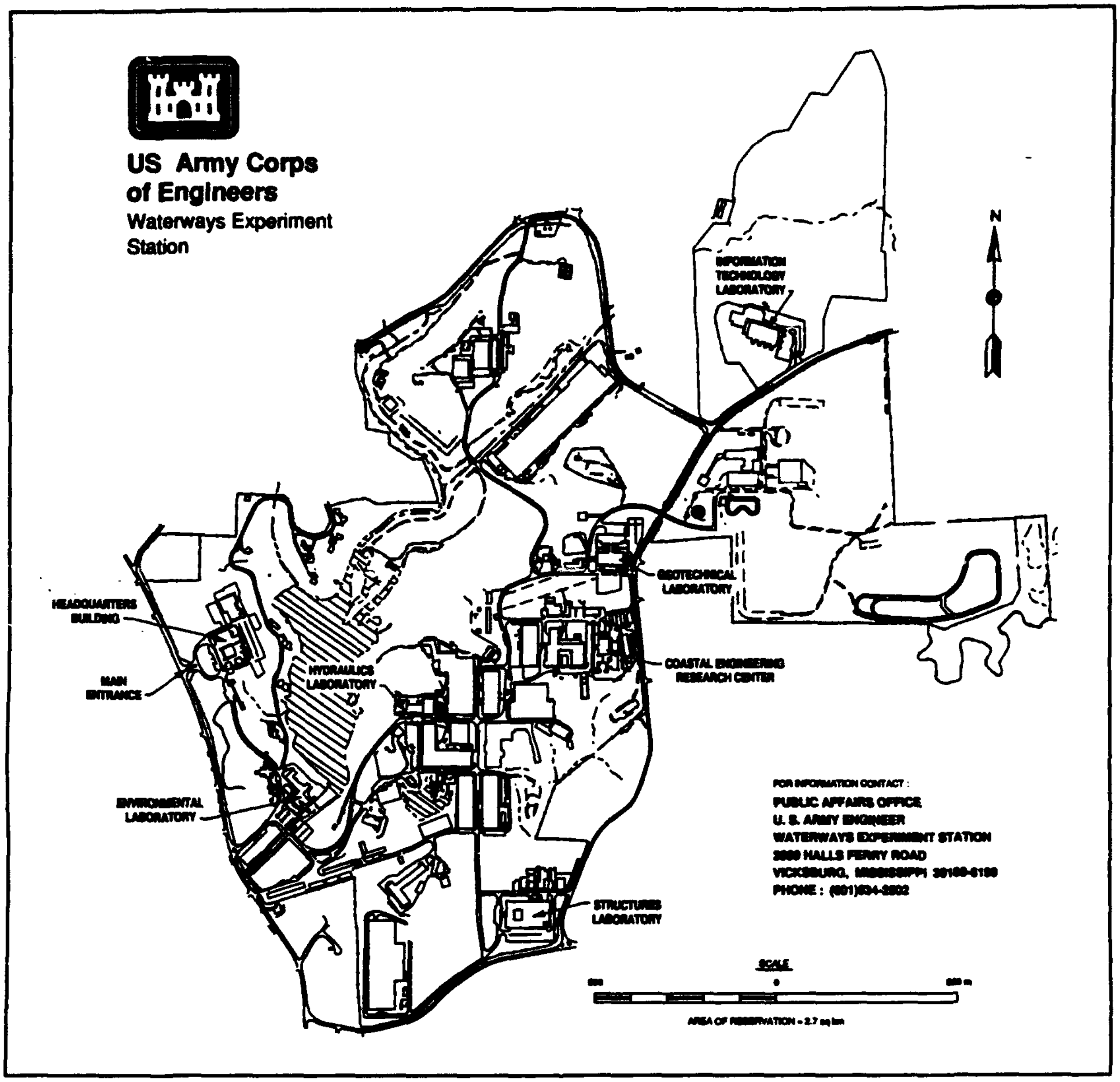

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Contents

Preface

The study reported herein was authorized by Headquarters, U.S. Army Corps of Engineers (HQUSACE), as part of the Geotechnical Problem Area of the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program. The work was performed under Civil Works Research Work Unit 32648, "Geomechanical Modeling for Stability of Existing Gravity Structures." The REMR Technical Monitor was Mr. Wayne Swartz (CECW-EG).

Mr. William N. Rushing (CERD-C) was the REMR Coordinator at the Directorate of Research and Development, HQUSACE. Mr. James F. Crews (CECW-O) and Dr. Tony C. Liu (CECW-EG) served as the REMR Overview Committee. The REMR Program Manager was Mr. William F. McCleese. U.S. Army Engineer Waterways Experiment Station (WES). Mr. Jerry S. Huie, Geotechnical Laboratory (GL), WES, was the Problem Area Leader.

The study was performed by Shannon and Wilson, Inc., under Contract No. DACW39-86-M-4062 to WES. Mr. Robert D. Bennett was Principal Investigator. This work was conducted under the direct supervision of Mr. Huie and under the general supervision of Dr. Don C. Banks, Chief, Soil and Rock Mechanics Division, GL. Dr. Paul F. Hadala was Assistant Director , GL, and Dr. William F. Marcuson III was Director, GL.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Bruce K. Howard, EN.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
degrees (angle)	0.01745329	radians
feet	0.3048	meters
pounds (force) per foot	14.5939	newtons per meter
pounds (force) per square foot	47.88026	pascals
pounds (force) per square inch	6.894757	kilopascals
pounds (mass) per cubic foot	16.01846	kilograms per cubic meter
square feet	0.09290304	square meters

1 Introduction

The following Phase II report for the U.S. Army Engineer Waterways Experiment Station supplements the initial report entitled, "Evaluation of Overturning Analysis for Concrete Structures on Rock Foundations" (Shannon and Wilson, Inc., in preparation). The purpose of the Phase I study was to review the overturning stability method presently used by the U.S. Army Corps of Engineers. Apparent short-comings in this method of analysis have resulted in computed safety factors of less than 1.0 for historically stable structures. The topics to be expanded upon in this report include the effects of strain compatibility and wall friction on the stability of rigid concrete gravity structures. These areas were singled out for additional review because they are ignored in present methods of analysis.

2 Summary of Findings

This report presents recommendations for modifications to the present Corps of Engineers' stability analysis for concrete gravity structures. These modifications are intended for use in conjunction with the current analysis procedures for a more accurate assessment of the factor of safety of existing structures. It is concluded from this study that the effects of wall friction and strain compatibility are significant when the foundation rock modulus is relatively low.

Strain Compatibility

The elastic rotation and translation of a rigid structure can be separated to obtain initial estimates of deformations produced by external moments and forces. Equations to compute these deformations are provided, and once determined, they can be used to estimate the reduction in earth pressure behind a wall that yields in the active sense. Although the equations are valid only for smooth, translating retaining walls, it has been shown that the influence of wall friction on earth pressures will be small and that a translating wall should produce similar reductions as does a rotating wall for the same deformation at the top of the wall.

No reasonable procedure is available for determining the partial passive pressure resistance of walls moving into a backfill. However, because usually only small heights of backfill are in front of a wall, this resistance may be ignored and at-rest earth pressures used.

Wall Friction

The works of Potyondy (1961) and Goh and Donald (1984) have presented reasonable estimates of maximum wall friction values that can develop as a function of the internal friction angle and cohesion of the backfill soil. These estimated values compare favorably to actual field and model test results at the limiting cases. For the present study, it is

sufficient to assume that the frictional resistance provided by the wall will be equivalent to the resistance through the soil on a vertical surface above the base of the structure.

The development of wall friction versus time is difficult to determine. From instrumented walls it appears that the development of the limiting friction angle is linear up to slippage between the wall and soil from an initial value. This initial value is dependent on the method of backfill placement and the height of the wall. A tentative correlation in determining mobilized wall friction is provided. It currently appears, based on instrumented case histories, that this resistance can be counted on for long-term effects. However, this point still needs consideration.

Case Study

The case study of a hypothetical concrete gravity structure is provided to show how the effects of strain compatibility and wall friction can be incorporated into the current Corps of Engineers' analysis procedures. It was concluded that the effect of these additional resistances is significant when the foundation rock modulus is low and significant deformation of the structure can occur.

3 Elastic Movements of Rigid Structures

From the theory of superposition, the solutions for elastic rotation and elastic translation can be used for determining backfill deformations for the overturning and sliding analyses, respectively. Whereas actual deformations may result from combined rotation and translation, the separation of these two forms of elastic movement will give conservative estimates of backfill deformations for the appropriate case of analysis.

Elastic Rotation

The elastic rotation of a rigid structure under an applied unit moment was presented in the Phase I report and is repeated here for completeness. The angle of rotation α is obtained from the solution below, where α is in radians:

$$\alpha = \frac{4m\,(1 - \mu^2)}{\pi E b^2} \tag{1}$$

where

m = Applied moment in units of moment per length

μ = Poisson's ratio

E = Young's modulus

b = Half width of base of structure

This solution was obtained from Muskhelishvili (1954) for the moment loading of a smooth, infinite strip on a semi-infinite mass (elastic half-space). It is redefined below for the general cases where the base may not be infinitely long:

$$\alpha = \frac{m\ (1 - \mu^2)\ I_\theta}{B^2 E} \tag{2}$$

where

B = Width of base (equal to 2b)

I_θ = Constant dependent on *L/B*

L = Length of base

It should be noted that for large *L/B* ratios, i.e., approaching an infinite strip, the solution defined above is equal to the solution from Muskhelishvili. The factor I_θ can be determined from the curve shown in Plate 1.

Elastic Translation

The translation of a rigid structure on an elastic medium acted upon by an applied horizontal force is obtained from Barkan (1962). This solution is not as rigorous as the solution for rotation. The average horizontal displacement d of the base is computed by assuming a uniform distribution of shear stress over the contact area and is given by:

$$d = \frac{P\ (1 - \mu^2)\ (L/B)^{1/2}}{B_x E} \tag{3}$$

where

P = Applied horizontal force in units of force per length

B_x = Constant dependent on μ and *L/B*

The factor B_x can be determined from the curve shown in Plate 2 for μ = 0.3.

Elastic Stresses Behind a Rigid Yielding Wall

Justification for the use of an elastic theory for stresses behind a yielding wall comes from the widespread use of the theory for various other analyses (settlement analysis, vertical pressure distribution, etc.). Finn (1963) published an article dealing with the development of stresses behind a rigid yielding retaining wall based on the theory of elasticity. Finn hoped to determine the pressure exerted by a soil mass behind a wall when

the boundary displacements were such that the classical plastic methods would not apply.

Finn's analysis was based on the classical concept of the "trap door" analysis, i.e., a soil mass with a yielding base. As the door is released away from the boundary, the pressure above it decreases due to soil arching. Finn applied this "trap door" analysis to that of a yielding retaining wall. He derived solutions for stresses developing behind rigid retaining walls rotating away from the backfill about their top and translating away from the backfill. He derived these solutions for cases where friction acts along the base (perfectly rough) and where it does not act along the base (perfectly smooth).

The pressure behind a smooth retaining wall that moves laterally away from the backfill was derived by Finn and is as follows:

$$\sigma_x = K_o \gamma z + \frac{4zh^2Ed}{(1 - \mu^2)\ \pi\ (z + h)^3\ (z - h)} \tag{4}$$

where

σ_x = Horizontal pressure at depth z

K_o = At-rest earth pressure

γ = Unit density

z = Depth from top of soil backfill

h = Height of backfill

E = Young's modulus of soil backfill

d = Horizontal displacement

μ = Poisson's ratio

This equation can be integrated to determine the net force P acting on the wall:

$$P = \Big|_o^z\ \frac{1}{2} K_o \gamma z^2 + \frac{Ed}{\pi(1 - \mu^2)} \left\{ \frac{1}{2} \ln \frac{\left|\frac{z}{h} - 1\right|}{\left(1 + \frac{z}{h}\right)} + \left[\frac{1}{\left(\frac{z}{h} + 1\right)} - \frac{1}{\left(\frac{z}{h} + 1\right)^2} \right] \right\} \tag{5}$$

The resultant of this force can be determined upon further integration and is stated below:

$$\bar{Y} = \frac{\sigma'_z \frac{1}{3} K_o \gamma z^3 + \frac{hEd}{\pi(1 - \mu^2)} \left\{ \frac{1}{2} \ln \frac{\frac{z}{h} - 1}{(1 + \frac{z}{h})} + \left[2 - \frac{3}{(1 + \frac{z}{h})} + \frac{1}{\left(\frac{z}{h} + 1\right)^2} \right] \right\}}{P} \qquad (6)$$

where

Z/h = Ratio of the depth at which the pressure behind the wall becomes negative to the height of the soil backfill (Appendix A, Sheet 6)

A stress singularity (division by zero, see Equation 5) develops at the point $z = h$ where infinite tension develops in the backfill. However, because the backfill cannot take a net tension, the stresses are assumed to go to zero below the depth where the tension adjustment to the earth pressure exceeds the at-rest pressure. It should be noted that the above equations are valid only for a backfill with a horizontal top surface and in which the backfill is either dry or completely submerged. It should be further pointed out that this equation is valid only for small displacements as is usually understood in the theory of elasticity such that slip surfaces will not occur.

Results from Finn's analysis generally show that when using elastic pressure theory, the pressure distribution against the wall departs from its linear shape with increasing deformation such that pressure shifts from the bottom of the wall toward the center of the wall. This shift produces a rise in the center of pressure above the lower one-third point. Finn's findings are confirmed by results from both Terzaghi (1934) and Sherif, Ishibashi, and Lee (1982) which show the center of pressure to be between 0.40 and 0.44 times the height of the wall above the base at the active state.

The equations shown above are for the case of a perfectly smooth wall translating away from the backfill. The results using a rough wall are similar but will result in lower earth pressures. Although, later in this report, wall friction will be shown to vary with deformation, its incorporation into this model would present complexities that are not easily resolved. Therefore, assuming a smooth wall will suffice for this study.

The model proposed by Finn is based on wall translation away from the backfill. Finn proposes no method to compute stresses when a wall rotates about its base. However, studies by Sherif, Ishibashi, and Lee (1982) and similar results by Terzaghi (1934), as well as finite element analyses by Clough and Duncan (1971), have shown that results produced by rotation about the base are similar to results where translation occurs for equal values of deformation at the top of the wall.

Development of Passive Pressures in the Elastic Range

Finn's work was restricted to wall movement in the active sense. A literature search was unsuccessful in finding a method that could easily provide theoretical solutions for partial passive resistance. There appears to be no satisfactory method available to compute passive pressure resistance as a function of deformation.

Based on model tests and finite element studies, it appears that a significant amount of passive resistance is mobilized by small deformations (i.e., deformations sufficient to place the outer backfill into the active state). Results from Terzaghi on model retaining walls and finite element analyses by Clough and Duncan show that, in general, passive pressures are mobilized at a much faster rate than active pressures for the same wall deformation. Thus, this resistance may prove more beneficial in increasing apparent stability where thick backfills are present than the reduction of pressure on the active side. Future studies may be able to determine a satisfactory elastic method for computing passive pressures.

4 Wall Friction

Wall friction is mobilized any time shear deformations occur along the contact between the wall and soil. This deformation occurs either through relative displacement between the wall and soil backfill (in the case of a rigid foundation this would imply settlement of the backfill) or through lateral earth pressures inducing movement of the wall. The frictional resistance that develops as a result of this displacement is dependent on the type of backfill, the nature of the wall, and the amount of wall movement.

Interface Friction

The soil-structure interaction behavior between various soil types and construction materials can be regarded as the initial step in understanding the role of wall friction on the stability of concrete gravity structures. The development of skin frictional resistance between a wall and soil backfill is excluded from the present Corps of Engineers' stability analysis. While skin friction is generally acknowledged as developing during shear deformations along this contact, there is some question to the long-term benefit of such a shearing force. Whether this force tends to dissipate with time, as a result of the creep of cohesive soils or vibrational effects on cohesionless soils, is currently unclear. Thus, for conservative design analyses, this resistance is ignored in practice.

Even if it is possible to verify that friction is sustained for long periods of time and during vibrations or cyclic loadings, the amount of wall deformation necessary to fully mobilize this force must be determined. Theoretically, the fully mobilized force should correspond to the initiation of slippage between the backfill soil and wall. For typical rigid gravity structures founded on a competent rock foundation, it can be shown that elastic deformations may be quite small. Thus, the wall friction may not be fully mobilized, and it becomes necessary to develop a relationship between the degree of wall movement and the mobilization of wall friction.

Previous investigators have generally incorporated the use of friction between soil or rock and construction materials as developing as a percentage of the soil's or rock's shear strength. Along rough contacts, such as

the base of concrete structures where concrete is cast directly against the soil or rock, the sliding surface is acknowledged to occur just below the base through the soil or on a joint just below the surface of the rock. This is the case because the concrete enters the irregularities of the soil or rock making this boundary surface stronger. Because the failure surface occurs below the base, the full friction angle or cohesion of the soil or rock discontinuity is developed. The same does not hold true for backfill soils placed against vertical concrete walls that are usually constructed by forms. In this instance, the wall is relatively smooth and provides less shearing resistance than a failure through the backfill material. Thus, the weakest link is along this contact, and the vertical friction angle or cohesion will be less than the maximum values the soil could develop.

For retaining wall design, Huntington (1957) suggests using an angle of wall friction δ equal to two-thirds of the friction angle ϕ and a wall adhesion value c_A equal to two-thirds of the soil cohesion c for analysis at the limiting equilibrium active and passive states. These values, although widely neglected for conservatism, have been accepted for design.

Potyondy (1961) performed a series of shear tests in both strain- and stress-controlled shear boxes to determine the skin friction between various construction materials and soil backfill types. Potyondy reported the ratio of the angle of wall friction to the soil's angle of internal friction (δ/ϕ) and the ratio of wall adhesion to soil cohesion (c_A/c). Unfortunately, there was no mention of the amount of shear deformation necessary to mobilize these values nor of the long-term effects of this friction.

Goh and Donald (1984) expanded upon Potyondy's testing. They felt that the direct shear device used by Potyondy contained numerous shortcomings in determining the value of skin friction and therefore modified the NGI Direct Simple Shear Device to overcome these problems. The new device was also able to reproduce the approximate shear mechanism behind a retaining wall and the approximate compaction procedures that would develop in the field. The results of these tests were fairly consistent to the δ/ϕ vales obtained by Potyondy. However, the values of c_A/c were approximately twice as high in the latter study for the cohesive materials. The authors attributed this difference to the compaction of the cohesive material against the concrete resulting in higher adhesive strengths.

A summary of the results of the above-mentioned tests for different backfill soils against concrete walls, in terms of the ratio of maximum developed interface friction to soil friction (at slippage), is presented in Table 1.

These results were compared with results from model retaining walls at the active and passive states. Terzaghi (1934) obtained δ/ϕ values for dry sand against a model wall that ranged from 0.65 to 0.85 at the active state for four different testing conditions. Fukuoka et al. (1977) obtained a value of δ/ϕ equal to 0.8 for sand against a large-scale concrete wall that yielded into the active state. Sherif, Ishibashi, and Lee (1982) obtained a

Table 1 Wall Friction Values Between concrete Structures and Various soil Types (after Goh and Donald 1984)		
Soil Type	**δ/ϕ**	**C_A/C**
Sand	0.8	—
Uncompacted silt	0.5	—
Compacted silt	0.8	—
Non-swelling clay	0.7	0.4
Uncompacted cohesive granular	0.4	0.4
Compacted cohesive granular	0.6	0.8

value of δ/ϕ of approximately 0.65 for a sand backfill that had developed its active earth pressure against an aluminum wall (estimated range of δ/ϕ from Potyondy for aluminum of 0.54 to 0.76). For a cohesive granular soil against a steel retaining wall, Fukuoka et al. (1977) obtained maximum wall friction values at the limiting active condition that resulted from a combination of friction and cohesion. Although no attempt was made to separate the individual contributions, the total force was found to be in the range that would be estimated from Potyondy's values for steel (δ/ϕ = 0.40 to 0.65, c_A/c = 0 to 0.35). The results of the aforementioned investigators suggest that the testings performed by Potyondy (1961) and Goh and Donald (1984) are reasonable for maximum wall friction values which will develop at the active state for various types of construction materials. Therefore, the values presented in Table 1 are proposed as maximum friction values that can be mobilized against concrete walls.

Deformation Condition

Fukuoka et al. (1977) concluded from a 2-year study of an instrumented retaining wall that "the existence of wall friction is so large that it could not be neglected in design." In order to rely on the full effects of wall frictional resistance, however, sufficient deformation of the backfill must occur. As the soil stretches during movement of the structure, the wall friction increases from an initial value to its maximum value. This maximum value will develop at or before the initiation of slippage between the backfill and the wall at the limiting equilibrium cases (i.e. active and passive states). The initial value of wall friction is difficult to determine as it depends on the properties of the backfill, the nature of placement (e.g., loosely placed as in siltation as compared with densely placed by compaction) and the wall material.

Results from Terzaghi (1934) and Sherif, Ishibashi, and Lee (1982) on rigid retaining wall models resting on solid foundations and yielding in the active direction show that a substantial initial value of wall friction may be dependent on the method of backfill placement behind the wall. For sands placed loosely behind a wall, Terzaghi presented experimental data that showed the initial value of tan δ is slightly less than the maximum value which it can attain. For dense sands, where density was achieved by a heavy compactive effort, the initial value of tan δ approached zero. Fukuoka et al. (1977) obtained similar results for both loose and compacted dense sand behind a yielding rigid wall. Sherif, Ishibashi, and Lee (1982) obtained an initial value of tan δ that was slightly less than one-half of its maximum value from using a shaking table to obtain high densities of the sand.

The results above may be explainable in terms of the effects of the compactive effort on the values of δ, ϕ, and K. The initial value of tan δ will usually decrease as a result of the compaction procedure for small walls. Compaction of backfill against a rigid wall will lock in additional horizontal stresses (as a function of the compaction-induced vertical stresses) that do not relax after the vertical stresses are removed. This is especially true for small walls (less than 25 ft[1] in height) where the compaction effort will have a substantial influence over the entire height of the wall. For high walls (greater than 50 ft in height), the horizontal stresses from the backfill overburden will be higher than the induced compaction stresses over much of the wall and thus control the value of the wall friction, i.e., the compaction-induced stresses will have a smaller influence over the height of the wall.

As the horizontal earth pressure increases from locked in compaction stresses δ tends to decrease if all other factors remain the same, thus, reducing the value of tan δ before earth pressure-related movement. This argument is strengthened by the results of Sherif, Ishibashi, and Lee (1982) with tests using sand that was densified rather than compacted. In this case, the locked in horizontal stresses are expected to be less significant because a shaking table was used rather than heavy compaction equipment. Thus, the initial value of tan δ is expected to be lower and in this instance approached one-half of its maximum value.

The above-mentioned investigations also show that, in general, the value of tan δ increases linearly with wall deformation towards its maximum value at or slightly before the active state is reached. Terzaghi's (1934) limited data on movement toward the passive state show that this movement has a small effect on the wall frictional resistance, and it is, therefore, negligible.

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[1] A table of factors for converting non-SI units of measurement to SI units is presented on page vi.

In summary, the initial value of the wall friction cannot be determined accurately. However, from literature it appears that the tangent of the wall friction angle generally increases linearly with increasing deformation of the wall for cohesionless soils. Thus, it is reasonable to account for some frictional resistance by assuming that tan δ varies linearly from an initial value to its maximum value (determined as some percentage of ϕ) as a function of the amount of wall movement necessary to develop the limiting equilibrium active case. Plate 3 is proposed, which takes into account friction angle development as a function of wall height and compactive effort.

Only limited data are available on wall friction values of cohesive soils; however, there is reason to believe that similar conditions exist. Gould (1970) presented test results from numerous investigators that showed an increase in the at-rest earth pressure with overconsolidation pressure resulting from compaction-induced stresses. Thus, the wall friction value would be expected to decrease with the overconsolidation ratio as the earth pressure increases. However, to be conservative, because of only limited experimental data, the wall friction (or adhesion) should be varied linearly from zero at the at-rest condition to its maximum values at the active state regardless of the initial placement conditions.

Long-Term Effects

The uncertainty of the long-term effects of wall friction subjects this type of resistance to scrutiny. Terzaghi's early results showed that at intermediate states (during displacement of the wall), if the wall was held constant, there was almost invariably a decrease of the coefficient of wall friction and an increase in the horizontal earth pressure. This phenomenon was explained in terms of the rotation of individual grains into less favorable positions such that the force required to keep them in this position decreases. In other words, the frictional resistance decreases, thereby increasing the net vertical reaction. Because the horizontal earth pressure is directly related to the vertical force, it also increases. Thus, as reasoned earlier, the wall friction should decrease. This result may be one reason for the current disregard of any wall frictional component. The decreases Terzaghi referred to, however, were in general relatively small. Furthermore, model wall tests since have demonstrated sustained friction forces.

Instrumentation of the Port Allen Locks (Kaufman and Sherman 1964) led the authors to the conclusion that substantial wall friction was available at small deflections. Although not actually measured, the wall friction was essential in obtaining realistic moment computations in back-calculating forces. These forces were generated by filling and emptying the locks and were measured through pressure cells installed on the lock walls. Frictional forces were believed to exist over the 2 years in which continuous monitoring was employed. Fukuoka et al. (1977) constructed a rigid cantilever retaining wall and backfilled it with clayey soil.

For over 2 years, this retaining wall was monitored. Twice, the wall was allowed to stand undisturbed for 9 months and 1 year, respectively. Both times, the load cells measured sustained frictional values. As a matter of fact, the frictional forces increased slightly throughout this period, most likely as a result of backfill settlement.

Frictional forces are also used for other applications. As explained in the Phase I report (Shannon and Wilson, Inc., in preparation), footing foundations on clay and sand are designed with a factor of safety relative to shear strength with no concern that these stresses will dissipate with time. Also, friction piles are designed with long-term friction components of resistance.

5 Proposed Changes to Methods of Analysis

It is proposed that the stability of concrete gravity structures be analyzed using earth pressures computed based on elastic theory and that vertical friction between the soil and concrete also be included. Rotation and translation may be computed based on earth pressure at rest as a first approximation. The earth pressures are then computed using the deformations from above and the equations developed by Finn (Part 3.13). The deformations may then be recomputed for the revised earth pressure distribution. Next, the amount of wall friction mobilized is estimated based on the deformations and the relationships proposed in Plate 3. With these forces defined, the rigid body equilibrium of the structure may be analyzed. An example calculation is provided in Appendix A and a discussion follows.

6 Practical Application of Suggested Methods

Appendix A presents example calculations for a typical concrete gravity structure (Plate 4) using the recommendations stated previously. Plate 5 shows the forces acting on this structure. Assumptions for the soil and rock properties are presented on page A1. Pages A2 and A3 show the external forces and moments acting on the wall. Page A4 computes the elastic rotation and translation of the structure from the influence of the forces and moments. Note that these deflections would be considered tolerable. Sherif, Fang, and Sherif (1984) presented results which showed that the active earth pressure was obtained, regardless of density or internal angle of friction of the backfill soil, at a horizontal translation of 0.001 times the height of the wall. From page A4 it can be seen that the *movement of the structure is less than this value.* Therefore, slip should not have occurred, and the deformation is small enough to use the theory of elasticity.

Page A6 shows the earth pressure distribution (no water pressure shown) for the elastic movements calculated above. For conservatism the smaller value of the elastic deformation due to rotation and translation was used. Also shown on this graph are the linear distributions for the at-rest earth pressure and the active earth pressure, with no friction acting along the wall. The distribution is seen to be non-linear and comparable to results of Taylor (1948) and Terzaghi (1934). The resultant force and location can be calculated based on equations derived from Finn and are presented on pages A7 and A8. Two interesting points should be made. First of all, the resultant earth pressure of 21,550 plf is between the active and at-rest pressures calculated, which is expected. Second, the center of pressure is located at 0.36 times the height of the wall. This finding confirms results of Terzaghi (1934) and Sherif, Ishibashi, and Lee (1982), which showed the center of pressure of the active condition to be above the one-third point, acting at 0.40 to 0.44 times the height of the wall (the center of pressure for the example is lower than these values because the full active condition has not been reached). It should be noted that the resultant force acting on the wall and its location were obtained through integration from the top of the wall down to 39.5 ft where the pressure behind the wall becomes zero.

If further calculations are made, it can be shown that the resultant earth force decreases non-linearly with increasing deformation of the wall away from the backfill. This non-linear variation is such that the largest force decreases occur under very small movements, after which only small changes in the earthen force are experienced with further deformation of the backfill up to the active state. This is consistent with the results of Terzaghi (1934) and Clough and Duncan (1971). It can also be shown that the center of pressure for the above case will increase to approximately 0.40 times the wall height if deformations of 0.001 times the height of the wall occur, the resultant force will be approximately equal to the active earth force as computed by the Coulomb theory for no wall friction. It should be noted, however, that large deformations such as these are beyond the scope of the elastic realm.

Page A9 presents a summary of the procedure suggested to determine the friction acting along the wall. Note that it is necessary to compute initial forces, moments, and deformations before calculating approximate wall friction values. From Plate 3, the mobilized wall friction value can be determined based on the suggested δ/ϕ value from Table 1. In this case only 53 percent of the tangent of the wall friction angle is mobilized. After computing an initial wall friction value, it is necessary to recompute the forces and moments acting on the wall with the incorporated wall friction force. Pages A10 and A11 present a summary of these calculations. Also, the elastic movements must be recomputed. Note that the change in the elastic deformation for rotation is substantially smaller than the change in deformation for translation. This is because of the long moment arm effect of the wall frictional force. Note, however, that the design deformation is only slightly less than that used previously because the smaller of the two deformations (from rotation and translation) are used. For structure and loading configurations in which the translation deformation is the larger component, a more significant change in the design deformation will be seen.

Pages A12 through A15 present calculations for the factor of safety against overturning (in terms of the location of the base resultant) and sliding (by using the limit equilibrium method of the Corps of Engineers, ETL 1110-2-256) using current Corps of Engineers' procedures and by modifying these procedures with the use of strain compatibility effects and a wall friction force. This work is summarized in the table on page A16. Also shown are the tiedown forces needed for a tiedown installed 5 ft from the backfill edge at the top of the wall at a 20-deg angle in the direction of the backfill. Note that the tiedown forces needed using both cases are approximately the same for the sliding analysis. However, there is up to a 30-percent change in the required remedial force in meeting overturning requirements. Once again, this effect is a direct result of the long moment arm for the friction force. It can be concluded that for overturning analysis, wall friction may have a significant influence on the location of the base resultant and remedial forces needed to meet Corps of Engineers' criteria.

As a final point, it should be emphasized that the example calculations in the appendix were based on a Young's modulus for the foundation of 100,000 psi, which is relatively low for a rock foundation. This low value could indicate highly fractured rock or a very weak intact rock (possibly weathered). For hard intact rock, modulus values of over 100 times the value used in the example would be expected, and thus all elastic movements would be negligible.

7 Future Studies

The development of friction or adhesion between a cohesive backfill soil and wall should be investigated further. It is probable that a substantial wall adhesion value is available, especially for normally consolidated material, which can increase apparent stability at movements that are not sufficient to develop the full active pressure condition. Also, long-term effects of wall friction should be researched to confirm that they exist in the field. Finally, the development of wall friction on curved surfaces or on surfaces that are not vertical above the base should be investigated to determine if higher resistances along these surfaces exist.

Future studies should also consider the development of passive earth pressures, as these pressures would provide substantially higher resistances for equal values of backfill height and backfill deformation than do the active pressures. Because backfills on the front of a wall are usually small or nonexistent, ignoring this development is usually insignificant.

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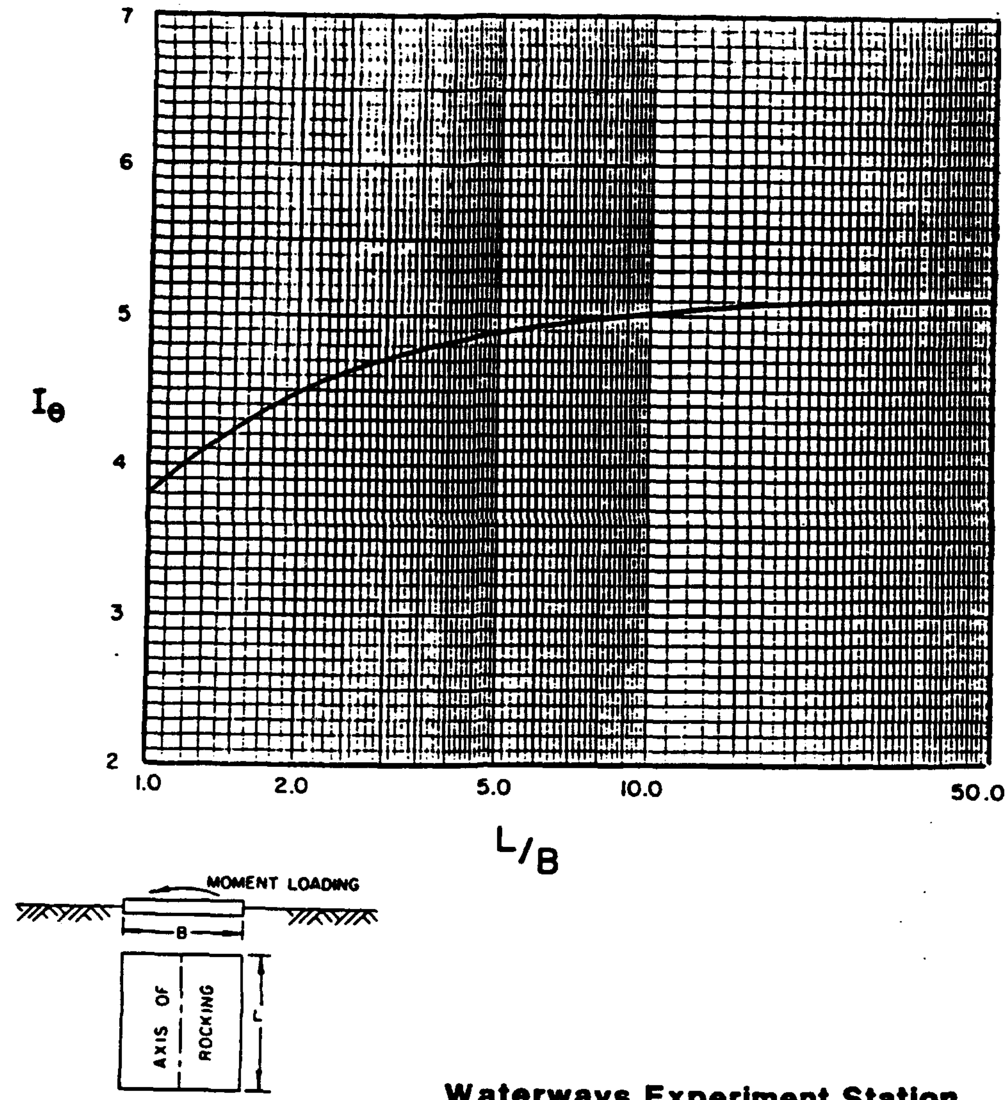

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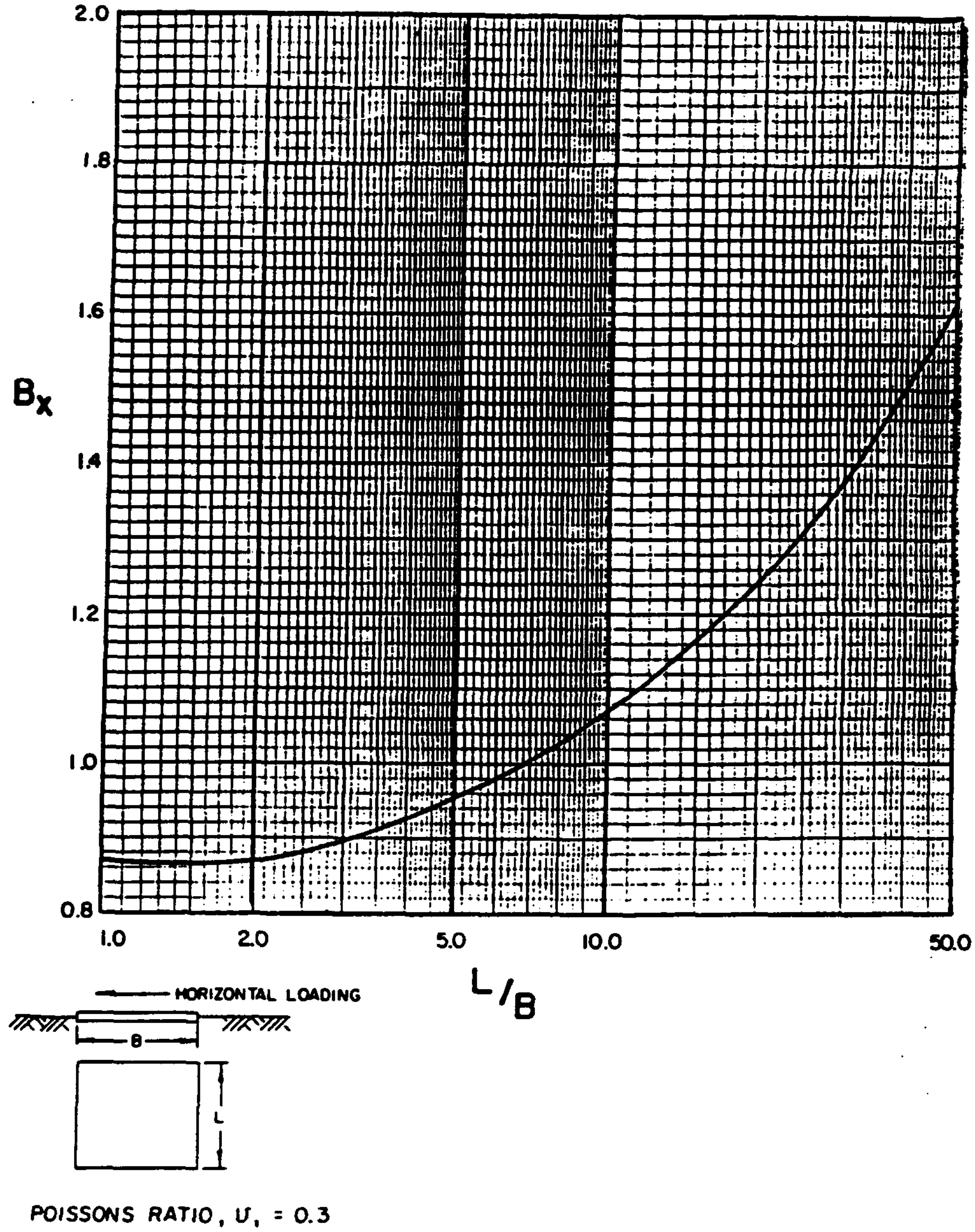
COEFFICIENT I_θ for RIGID RECTANGULAR FOOTING



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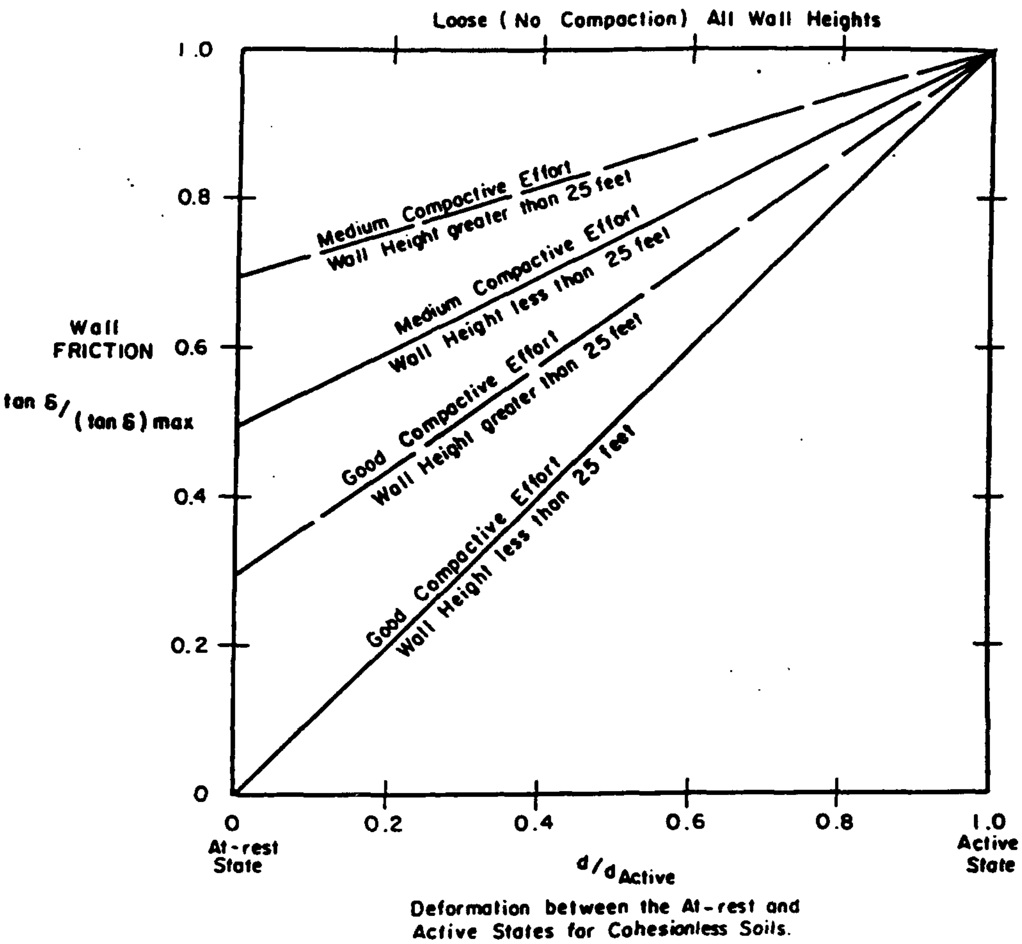
Phase II Studies
COEFFICIENT B_x for RIGID RECTANGULAR FOOTING



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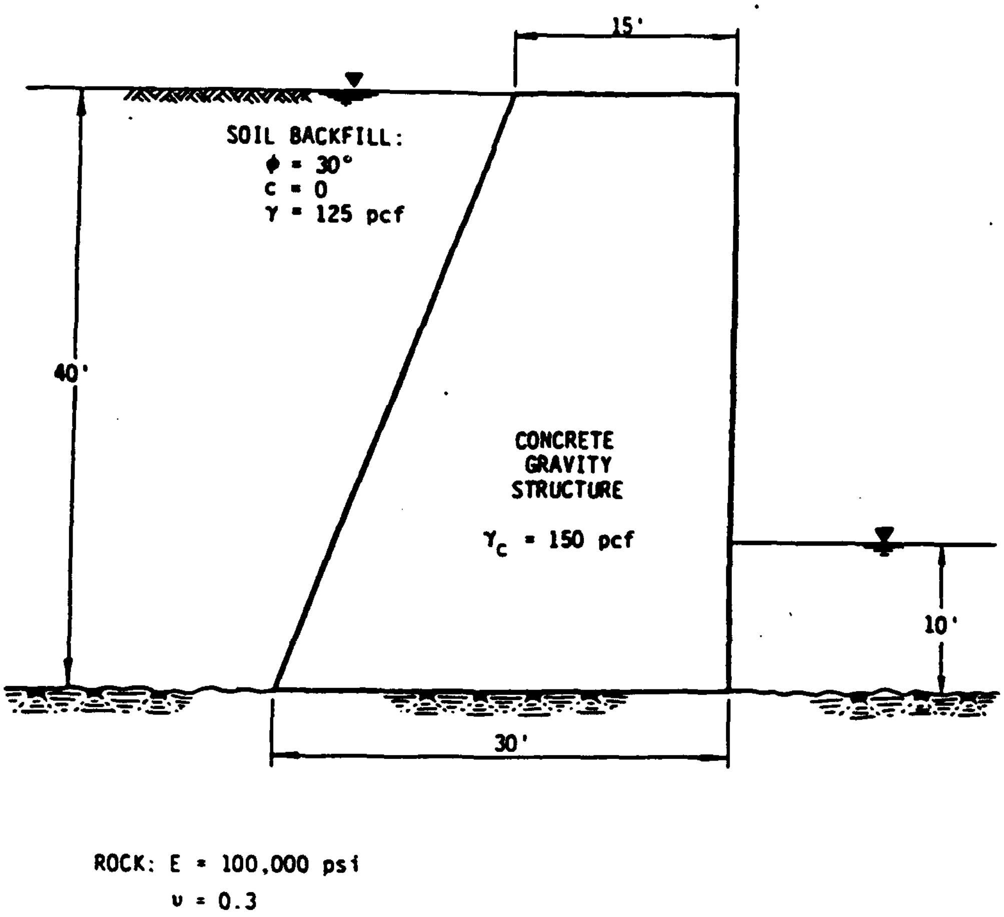
Proposed Correlation Between Wall Deformation and the Development of Wall Friction



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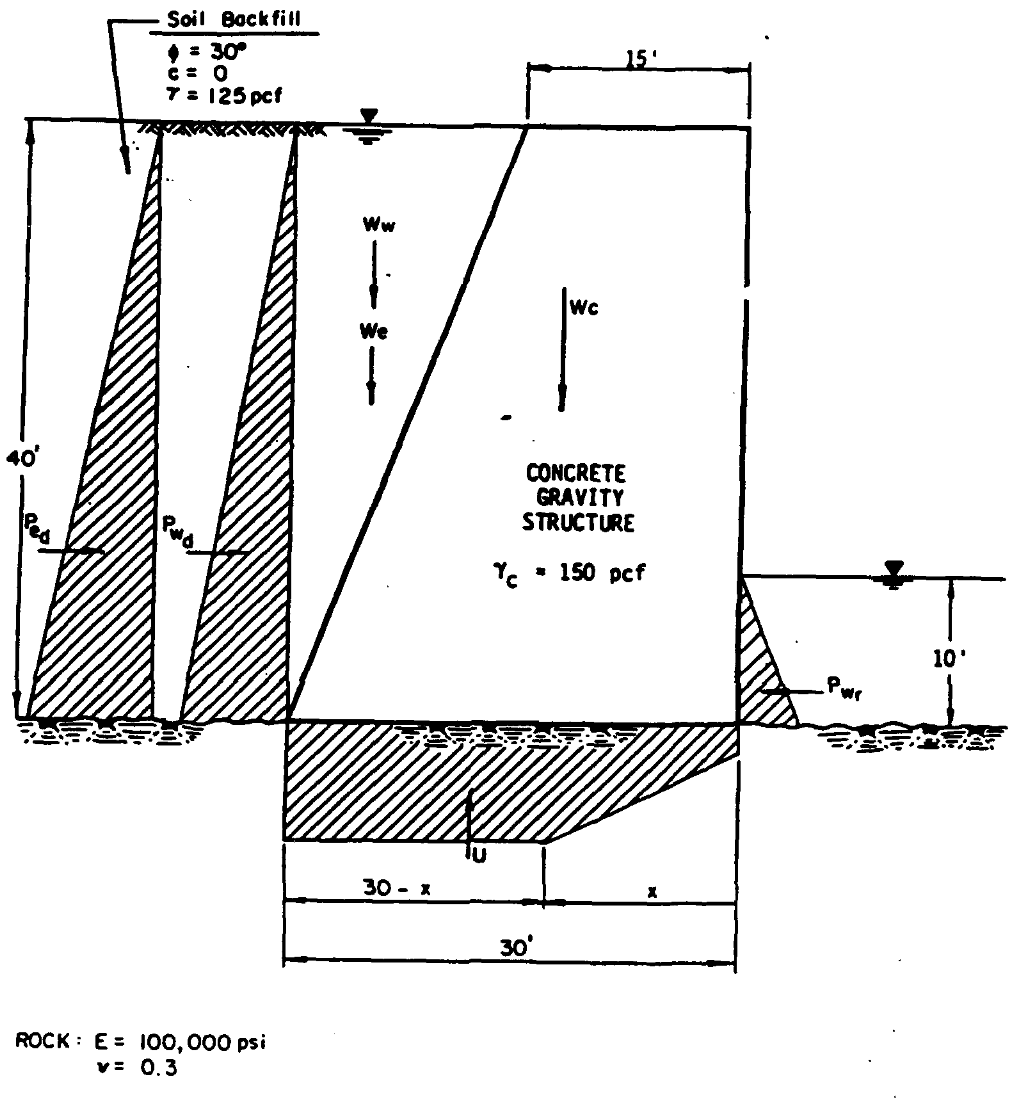
TYPICAL GRAVITY STRUCTURE CONFIGURATION



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External Forces Acting
on a Gravity Structure

Appendix A
Sample Calculations

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Sample Calculations for a Typical Concrete Gravity Structure using the Recommendations Stated in the Report

1) Assumptions

- See Plate 4 for typical configuration

- Backfill sand

$\gamma = 125 \text{ pcf}$
$\phi = 30°$ *(Medium Compactive Effort)*
$c = 0$

- Modulus of backfill soil computed at $2h/3 = 26.7'$

$E = 100 - 500 (\sigma'_3)$ *Peck, Hanson, Thornburn (1974, pg 86)*

use 300
$\sigma'_3 = K_o \gamma' z$

$K_o = 1 - \sin\phi = 1 - \sin 30° = 0.5$

$\sigma'_3 = 0.5 (125 - 62.4 \text{ pcf})(26.7 \text{ ft})$
$= 835.7 \text{ psf}$

$E = 300 (835.7 \text{ psf}) \approx 250,000 \text{ psf}$

- Poisson's Ratio

$K_o = 0.5 = \frac{\nu}{1-\nu}$

$\nu = 0.33$

- Modulus of foundation

$E = 100,000 \text{ psi} = 1.44 \times 10^7 \text{ psf}$
$\nu = 0.3$
$\phi_f = 30°$ (along joint)

- Length of Structure (or Monolith), L

300 feet

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2) Forces and Moments (refer to Plate 5 for notation)

$P_{w_a} = \frac{1}{2}(0.0624\ kcf)(40ft)^2 = 49.9\ klf$

$P_{E_0} = \frac{1}{2}(0.5)(0.0626\ kcf)(40ft)^2 = 25.0\ klf$

← Assume at-rest case initially

$P_{w_p} = \frac{1}{2}(0.0624\ kcf)(10ft)^2 = 3.1\ klf$

$W_w = \frac{1}{2}(0.0624\ kcf)(40ft)(15ft) = 18.7\ klf$

$W_E = \frac{1}{2}(0.0626\ kcf)(40ft)(15ft) = 18.8^{klf}$

$W_c = \frac{1}{2}(0.150\ kcf)(15+30)(40ft) = 135.0^{klf}$

$U = \frac{1}{2}(0.0624\ kcf)(10ft)x + \frac{1}{2}(0.0624\ kcf)(40ft)x + 0.0624\ kcf(40ft)(30-x)$

$= 0.312x + 1.248x + 74.88 - 2.496x$

$= 74.88 - 0.936x$

$M_{P_{w_a}} = 49.9\ klf(40/3) = 665.3^k$

$M_{P_{E_0}} = 25.0\ klf(40/3) = 333.3^k$ Assume acting at one third point initially

$M_{P_{w_p}} = 3.1\ klf(10/3) = 10.3^k$

$M_{W_w} = 18.7\ klf(15+10) = 467.5^k$

$M_{W_E} = 18.8^{klf}(15+10') = 470.0^k$

$M_{W_c} = (15)(40)(0.15)(15/2) + \frac{1}{2}(15)(40)(0.15)(15+15/3)$

$= 1575^k$

$M_U = 0.312x(x/3) + 1.248x(2x/3) + (74.88 - 2.496x)(x/2 + 15)$

$= 0.104x^2 + 0.832x^2 + 37.44x - 1.248x^2 + 1123.2 - 37.44x$

$= -0.312x^2 + 1123.2$

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3) Solve for x

$$\Sigma V = 18.7 + 18.8 + 135.0 - 79.88 + 0.936x = 97.62 + 0.936x$$

$$\Sigma M = -665.3 - 333.3 + 10.3 + 467.5 + 470.0 + 1575 + 0.312x^2 - 1123.2$$

$$= 401.0 + 0.312x^2$$

$$\Sigma M / \Sigma V = x/3 = \frac{401.0 + 0.312x^2}{97.62 + 0.936x}$$

$$1203 + 0.936x^2 = 97.62x + 0.936x^2$$

$$x = 12.32 \text{ feet}$$

$$\therefore \quad U = 74.88 - 0.936(12.32) = 63.3^{kips}$$

$$M_u = -0.312(12.32)^2 + 1123.2 = 1075.8^{k}$$

$$\Sigma V = 97.62 + 0.936(12.32) = 109.2^{kips}$$

$$\Sigma M = 401.0 + 0.312(12.32)^2 = 448.4^{k}$$

4). Solve for the external moment and horizontal force producing potential rotation and translation, respectively

$$\Sigma H = 49.9 + 25.0 - 3.1 = 71.8^{kips}$$

$$\Sigma M' = |\Sigma M - M_k| = 448.4 - 1575 = 1126.6^{k}$$

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5) Compute the elastic rotation of the structure

$$\alpha = \frac{m(1-\nu^2)I_\theta}{B^2E}$$

$$\alpha = \frac{1126.6^k(1-(0.3)^2)5.01}{(30ft)^2(1.44\times10^4 ksf)}$$

I_θ (for $L/B = 10$) = 5.01

$= 3.96\times10^{-4}$ rad

deformation at the top of wall = $(3.96\times10^{-4}$ rad$)(40$ ft$)$

= 0.0158 feet

6) Compute the elastic translation of the structure

$$\delta = \frac{P(1-\nu^2)(L/B)^{1/2}}{B_x E}$$

$$= \frac{71.8^{kef}(1-(0.3)^2)(10)^{1/2}}{1.070(1.44\times10^4 ksf)} = 0.0134 \text{ feet}$$

(B_x from Plate 2)

Use the smallest of the two deflections for conservatism

$\therefore \delta^* = 0.0134$ feet

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7) Compute the horizontal earth pressure against the retaining wall for a movement of 0.0134 feet.

$$\sigma_x = K_a \gamma z + \frac{4 d h^2 E z}{(1-\nu^2)\pi (z+h)^2 (z-h)}$$

from the property values on page 1

$$\sigma_x = 0.5(62.4pcf)z + \frac{4(0.0134ft)(40ft)^2(250,000psf)z}{(1-(0.33)^2)\pi(z+40)^2(z-40)}$$

$$\sigma_x = 31.3z + \frac{7.66\times10^6 z}{(z+40)^2(z-40)}$$

z	σ_x
0	0
5	145
10	290
15	440
20	590
25	735
30	870
35	970
38	880
39	615
39.5	30

σ_x goes to 0 @ approximately 39.5 feet

$K_a = \tan^2(45 - 30/2) = 1/3$

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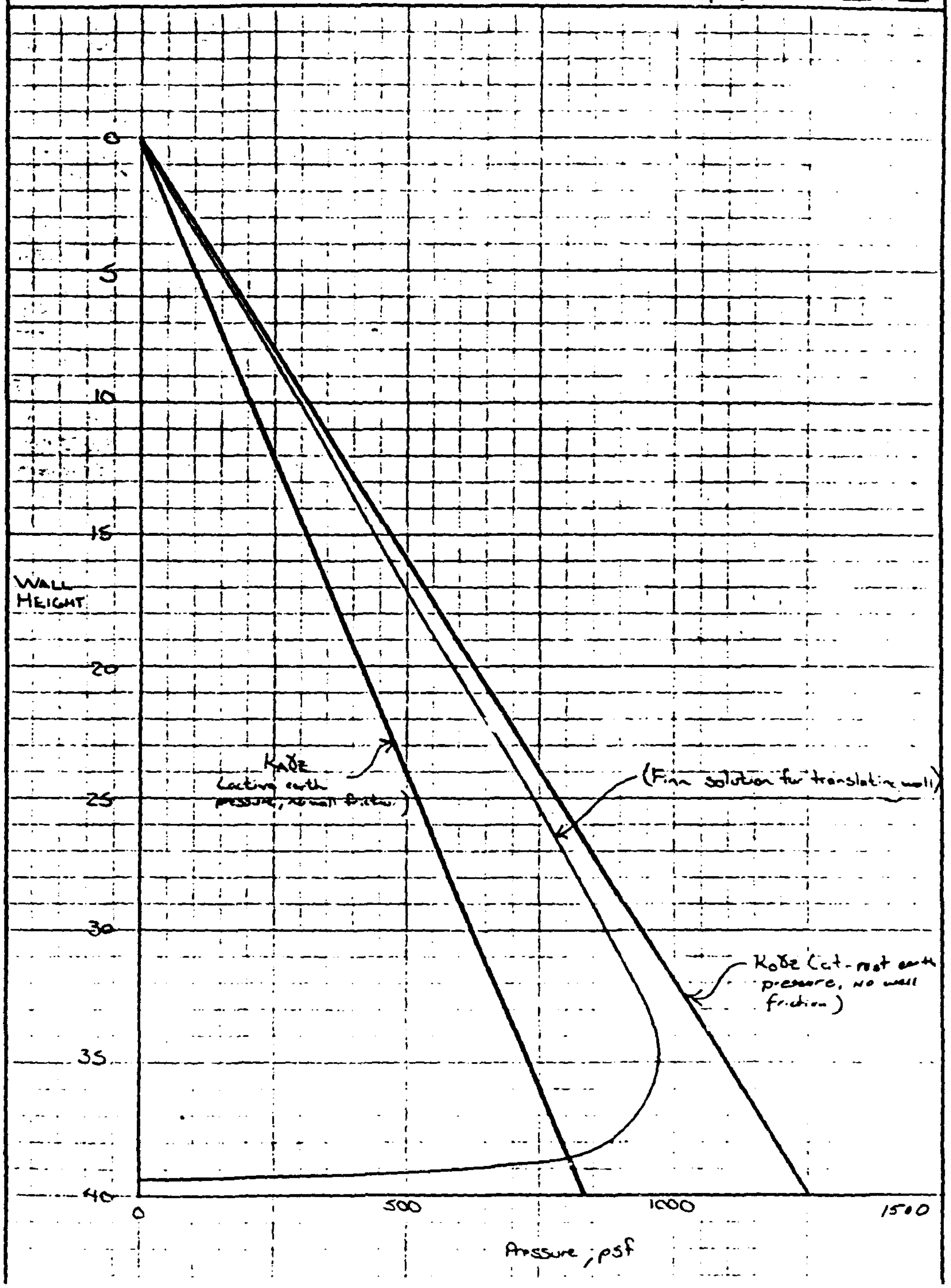

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Resultant earth pressure force

$$P = \left[\tfrac{1}{2} K_0 \gamma z^2 + \frac{E d}{\pi (1-\mu^2)} \left[\tfrac{1}{2} \ln \frac{|\frac{z}{h} - 1|}{(1 + \frac{z}{h})} + \left[\frac{1}{(\frac{z}{h}+1)} - \frac{1}{(\frac{z}{h}+1)^2} \right] \right] \right]_{z=0}^{z=39.5}$$

$$P = \tfrac{1}{2}(0.5)(62.6\,\text{pcf})(39.5)^2$$

$$+ \frac{(250{,}000\,\text{psf})(0.0134\,\text{ft})}{\pi\,(1 - (0.33)^2)} \left[\tfrac{1}{2} \ln \frac{|0.99 - 1|}{(1 + 0.99)} + \left[\frac{1}{(0.99+1)} - \frac{1}{(0.99+1)^2} \right] \right]$$

Note - earth pressure force must be greater than 0. Thus, where the earth pressure becomes negative it is considered 0 and $z/h = 39.5/40 = 0.99$

$$= 24{,}418 + 1197 \left[-2.6467 + 0.25 \right]$$

$$= 24{,}418 - 2869$$

$$= 21550 \text{ plf}$$

8) At-rest force from Coulomb solution = 25040 plf
Active force from Coulomb solution = 16,606.5 plf
Deformation / Height of wall = 3.35×10^{-4}
Intermediate force = 21,550 plf

Note the above intermediate force is greater than the active force indicating full deformation of the backfill has not occurred.
$\delta/H = 3.35 \times 10^{-4}$ which is slightly less than the value of 1.00×10^{-3} which is commonly taken as deformation needed for the active case.

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7) Center of pressure of previous case

$$\bar{y} = \frac{\frac{1}{3}k_o \gamma z^3 + \frac{hE\delta}{\pi(1-\nu^2)}\left[\frac{1}{2}\ln\frac{|\frac{z}{h}-1|}{(1+\frac{z}{h})} + \left(2 - \frac{3}{(1+\frac{z}{h})} + \frac{1}{(1+\frac{z}{h})^2}\right)\right]}{\text{Net Force}}$$

$$\bar{y} = \frac{\frac{1}{3}(0.5)(62.4)(39.5)^3 + \frac{(250,000)(40)(0.0134)}{\pi(1-0.33^2)}\left[\frac{1}{2}\ln\frac{0.01}{1.99} + \left(2 - \frac{3}{1.99} + \frac{1}{(1.99)^2}\right)\right]}{21,550}$$

$$\bar{y} = \frac{643,000 + 47866(-1.902)}{21,550}$$

$\bar{y} =$ 25.6 feet

$(1 - y/h) =$ 0.36 (acting point)

Note the location of resultant force is greater than 1/3h

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10) Find Friction acting along Wall ($\delta_{mobilized}$)

From Plate 3, using relative displacement of $3.35 \times 10^{-4} / 1 \times 10^{-3} = 0.335$

and medium compactive effort for sand for a wall > 25 feet high

$\tan\delta / \tan\delta_{max} = 0.79$

Maximum wall friction $\delta_{max}/\phi = 0.8$ from Potyondy

$\therefore \delta_{max} = 24°$, $\tan\delta_{max} = 0.445$

$\therefore \tan\delta / \tan\delta_{max} = 0.79$ so $\delta_{mobilized} = 19.4°$

11) WALL Friction Force, F

$F = P \tan\delta_{mob}$

$= 21550 \tan 19.4 = 7576$ plf downward

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12) Recompute the external forces and moments

$P_{wa} = 49.9^{kef}$ →

$P_{E} = 21.55^{kef}$ →

$P_{wp} = 3.1^{kef}$ ←

$W_{w} = 18.7^{kef}$ ↓

$W_{E} = 18.8^{kef}$ ↓

$W_{c} = 135.0^{kef}$ ↓

$F = 7.6^{kef}$ ↓

$U = 74.88 - 0.936x$ ↑

$M_{o} = 665.3^{k}$ ↻

$M_{P_E} = 310.3$ ↻

$M_{P_{wp}} = 10.3^{k}$ ↺

$M_{w_w} = 467.5^{k}$ ↺

$M_{w_E} = 470.0^{k}$ ↺

$M_{w_c} = 1575^{k}$ ↺

$M_{F} = 228^{k}$ ↺

$M_{u} = -0.312x^2 + 1123.2$ ↻

$$\frac{\Sigma M}{\Sigma V} = \frac{x}{3}$$

$$\frac{652.0 + 0.312x^2}{105.22 + 0.936x} = \frac{x}{3} \qquad x = 18.59 \text{ feet}$$

$\therefore U = 57.5$

$M_u = 1015.4$

$\Sigma V = 122.6$

$\Sigma M = 759.8$

$\Sigma H = 68.4^{kef}$

$\Sigma M' = 815.2^{k}$

$$\text{Elastic Rotation} = \alpha = \frac{815.2\,(1-0.3^2)\,5.01}{30^2\,(1.44 \times 10^4)} = 2.87 \times 10^{-4} \text{ rad}$$

$$\partial = 2.87 \times 10^{-4} \times 40 = 0.015 \text{ feet}$$

$$\text{Elastic Translation} = \partial = \frac{68.4\,(1-0.3^2)\,(10)^{1/2}}{1.070\,(1.44 \times 10^4)} = 0.0128 \text{ feet}$$

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Using smaller of two:

$\partial = 0.0115$

$\partial/H = 0.0115/40 = 2.87 \times 10^{-4}$

$\tan\delta / \tan\delta_{max} = 0.78$ from Plate 3

Resultant e.p. force

$P = 22{,}010$ plf

Location of resultant

$\bar{y} = 26.3$ feet

$1 - \bar{y}/h = 0.34$ ratio of location of resultant to wall height above base

Wall Friction Force

$\tan\delta/\tan\delta_{max} = 0.78$, $\tan\delta_{mob} = 0.78(0.445) = 0.227$

$\delta_{mob} = 19.1°$

$F = 22{,}010 \tan 19.1 = 7620$ plf downward

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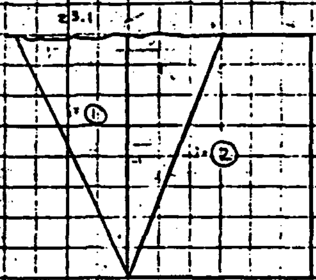
13) Compute the FS against overturning and sliding using present Corps method (see forces from page 2)

Overturning: Location of base reaction

$\Sigma M / \Sigma V = 448.4 / 109.2 = 4.1$ (outside 1/3 point)

Sliding: Limit Equilibrium Method



	i	α_i	L_i	H_{Li}	H_{Ri}	V_i	W_i	U_i	ΔP_i
try FS = 1.00	1	-60	46.2	0	0	0	57.8	57.8	-66.7
	2	0	30	0	3.1	←	109.2	→	66.1
									-0.6

FS = 1.0 (less than required 2.0)

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14) Compute the tiedown force needed for overturning resultant within middle third and $FS_{\text{slide}} = 2.0$

– Compute the tiedown force necessary for FS = 2.0, assume resultant is in kern (i.e. sliding controls)

Assume tiedown acts at 20°, 5' from backfill edge at top

FS=2.0	i	α	L	H	H_a	V	W	U	ΔP
	1	-53.05	50.05	0	0	0	75.3	62.5	-78.3
	2	0	30	0	3.1	⟵	x	⟶	y

y = 78.3 for FS = 2.0

if resultant is in kern, x = 30

$\Sigma V = 97.62 + 0.936(30) = 125.7$

$\Sigma M = 401.0 + 0.312(30)^2 = 681.8$

added increase to normal force by tiedown

$T\cos 20°$

added horiz resistance

$T\sin 20$

$$\left[(125.7) + T\cos 20\right]\frac{\tan 30}{2} - (0 - 3.1 - T\sin 20) = 78.3$$

$$36.3 + 0.27T + 3.1 + 0.34T = 78.3$$

$T = 63.8^{\text{kf}}$ force needed

– Compute T for $\Sigma m/\Sigma v = 10$ (kern)

$\Sigma M = 681.8 + T\cos 20(10) + T\sin 20(40)$

$\Sigma V = 125.7 + T\cos 20$

$$\frac{681.8 + 23.1T}{125.7 + 0.94T} = 10$$

$T = 41.9^{\text{kf}}$ needed

Sliding controls $T = 63.8^{\text{kf}}$

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15) Compute the FS against overturning and sliding using suggestions detailed in this report.

Overturning

$\Sigma M = 759.8$
$\Sigma V = 122.6$

$\Sigma M / \Sigma V = 6.2$ (outside kern)

Sliding

	i	α_i	L_i	H_{Li}	H_{Ri}	V_i	W_i	U_i	ΔP_i
try FS=1.25	1	-57.4	47.5	0	0	0	64	59.3	-70.5
	2	0	30	0	3.1	← 122.6 →			59.7
									-10.8
try FS=1.10	1	-58.8	46.7	0	0	0	60.5	58.3	-68.2
	2	0	30	0	3.1		122.6		67.4
									-0.8
try FS=1.05	1	-59.4	46.5	0	0	0	59	58.0	-67.4
	2	0	30	0	3.1		122.6		70.5
									+3.1

FS = 1.09

16) Compute the tiedown force for overturning resultant within middle third) and $FS_{LEM} = 2.0$

- Compute T for FS = 2.0 LEM (assume resultant is in kern)
Assume tiedown acts @ 20°, 5' from backfill edge @ top

	i	a	L	H_L	H_R	V	W	U	ΔP
FS = 2.0	1	-53.05	50.05	0	0	0	75.3	62.5	-78.3
	2	0	30	0	3.1		← x →		y

y = 78.3 for FS = 2.0

if resultant is in kern, x = 30

$\Sigma V = 105.22 + 0.936(30) = 133.3$

added increase to normal force
T cos 20

added increase to horizontal
T sin 20

$$\left[133.3 + T\cos 20\right]\frac{\tan 30}{2} - (0 \cdot 13.1 - T\sin 20) = 78.3$$

$0.61T = 36.72 \quad T = 60.2$ kpf

- Compute T for $\Sigma M / \Sigma V = 10$ (kern)

$\Sigma M = 652 + 0.312(30)^2 + T\cos 20(10) + T\sin 20(40)$
$\Sigma V = 105.22 + 0.936(30) + T\cos 20$

$\Sigma M = 932.8 + 23.1T$
$\Sigma V = 133.3 \quad 0.94T$

$$\Sigma M / \Sigma V = \frac{932.8 + 23.1T}{133.3 + 0.94T} = 10$$

$T = 29.2$ kpf

Sliding $T = 60.2$ kpf (controls)

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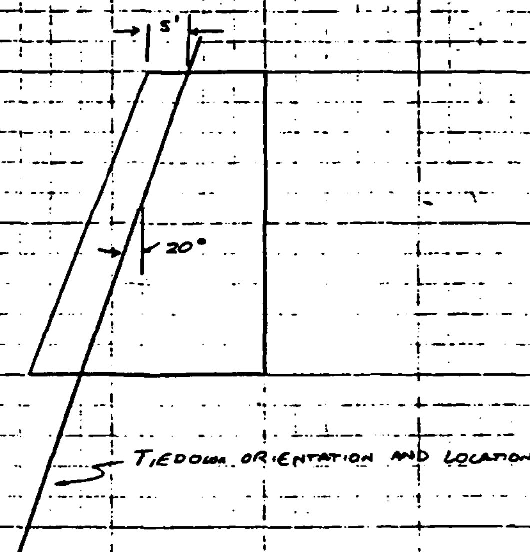
SUMMARY

METHOD	Location of Resultant	SLIDING F.O.S.	TIEDOWN FORCE [1]	TIEDOWN FORCE [2]
CORPS OF ENGINEERS	4.1 ft	1.00	63.8 kip	41.9 kip
CURRENT MODIFICATIONS	6.2 ft	1.09	60.2 kip	29.2 kip

1. Necessary for sliding F.O.S. = 2.0
2. Necessary for overturning $\Sigma M / \Sigma V = 10$



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This report presents recommendations for modifications to the present Corps of Engineers' st. bility analysis for concrete gravity structures. These modifications are intended for use in conjunction with the current analysis procedures for a more accurate assessment of the factor of safety of existing structures. It is concluded from this study that the effects of wall friction and strain compatibility are significant when the foundation rock modulus is relatively low.

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